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PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

JULY, 1951

PETENWELL HYDROELECTRIC PROJECT

By E. Montford Fucik, Assoc. M. ASCE

POWER DIVISION

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N.Y.

PRICE $0.50 PER COPY

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Proceedings—Vol. 77 July, 1951 Separate No. 81

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

PETENWELL HYDROELECTRIC PROJECT

By E. Montford Fucik,[1] Assoc. M. ASCE

Synopsis

The problem of constructing and operating a water-retaining structure of considerable size on sand foundations is described in this paper. The Petenwell Hydroelectric Project consists of a concrete powerhouse and spillway and several miles of earth dams and dikes, all founded on a deep bed of sand. The concrete structures were built adjacent to the river channel, and the river then was diverted to complete the earth dam.

The spillway sections were the result of model tests to produce a design that would avoid scouring of the downstream channel. The construction and details of these spillway sections are described, and several typical drawings are given. Various appurtenances necessary for successful operation of the project also are described.

General Description

The Petenwell Hydroelectric Project is located on the Wisconsin River, near Necedah, Wis., about 33 miles north of the famous Wisconsin Dells, as shown in Fig. 1. Construction was started in August, 1947, and the first unit was put into operation under reduced head on August 29, 1949. The reservoir was completely filled, and all units were operating in April, 1950.

The project is owned by the Wisconsin River Power Company, a company formed by the Consolidated Water Power and Paper Company, Wisconsin Rapids (Wis.), the Wisconsin Public Service Company, Green Bay (Wis.), and the Wisconsin Power and Light Company, Madison (Wis.).

The principal structures of the project, as shown in Fig. 2, are the concrete powerhouse and spillway flanked by approximately 9 miles of earth dams and dikes. The gross head is 42 ft with the reservoir at normal operating level and the full gate flow of 6,960 cu ft per sec through the units. The powerhouse contains four vertical, adjustable-blade Kaplan propellor turbines of 7,200 hp

Note—Written comments are invited for publication; the last discussion should be submitted by **January, 1951.**

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each at full gate, with generators rated at 6,250 kva, 0.8 power factor, three-phase 60 cycle. The speed of the units is 163.6 rpm.

Foundations

The foundation for the entire project consists of a deep bed of medium grained sand. Drilling has indicated this bed to be at least 100 ft deep with few, if any, lenses of silt or clay. As a result of these conditions, it was necessary to design all structures without attempting to provide any complete cutoff for leakage and to use the sand for all the earth structures. The nearest suitable material finer than sand was too far away to be used economically.

Fig. 1.—Petenwell Project Location Map

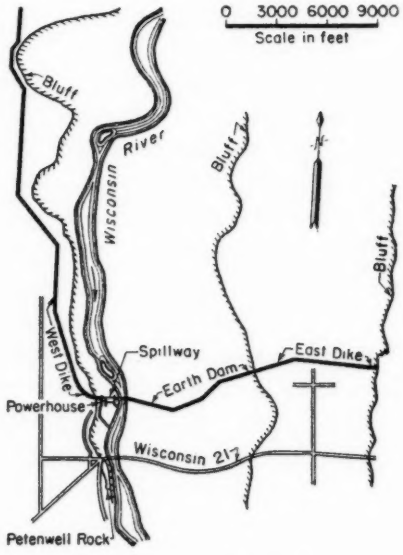

Fig. 2.—Plan of Project Site

Typical grain size curves for the foundation materials are shown in Fig. 3. It can be seen that there are very few fines in this material, with the result that a considerable quantity of leakage had to be anticipated under the structures and through the earth dams. Laboratory permeability tests made on this sand indicated permeability coefficients ranging from $k = 93$ ft per day to $k = 555$ ft per day for void ratios varying from 0.46 to 0.60, respectively.

The natural density of the sand in the spillway and powerhouse foundations was found to vary from 95 lb per cu ft to 112 lb per cu ft.

Arrangement of Structures

General Construction.—The powerhouse and spillway were constructed on the west bank of the river without encroaching on the normal river channel. This method of development saved the expense of elaborate cofferdams in the river and considerably reduced the pumping problem during construction.

All the material excavated for the tailrace back to the normal river course was used in the construction of the dams. Closure of the dam was made at the river section, while river diversion was accomplished by omitting the gravity (ogee) portion of the regulating bay of the spillway. Final closure was made by putting stop logs in front of the regulating bay and completing the ogee section behind these stop logs.

This plan of construction took full advantage of the sand foundations by permitting the powerhouse and spillway construction to proceed without requiring any work to be done in the normal river channel until closure of the dam. The only cofferdams required were an inexpensive sand fill around the area (cast up from, and a by-product of, the excavation) to protect against floods that would cause the river to overflow its banks, and a short crib cofferdam in front of the powerhouse. The latter cofferdam was necessary to protect the powerhouse work after diversion had been made because powerhouse construction was delayed by slow delivery of reinforcing steel.

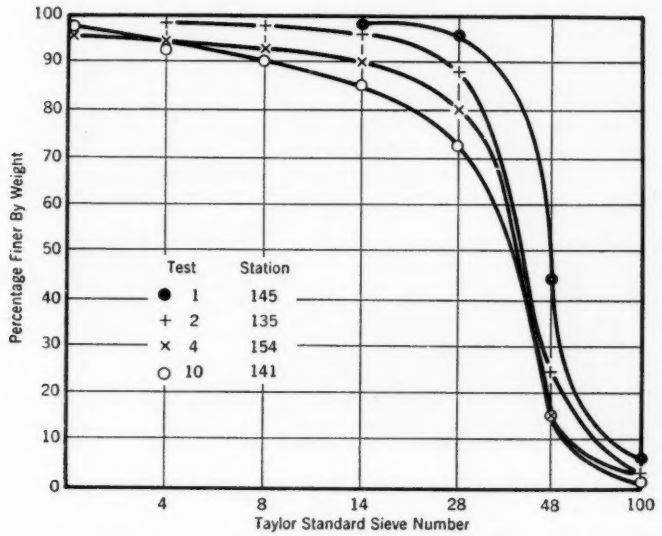

FIG. 3.—FOUNDATION MATERIAL GRAIN SIZE DISTRIBUTION

When the spillway was completed up to the crest level and the earth dams were at sufficient height, the river closure was made. This was done by building the earth fills up on each side of the river and then bulldozing the fill into the river channel from each side to close the gap in the middle. Riprap in sizes up to about 24 in. was used to aid in the initial closure made at the downstream toe of the dam. About 2,000 to 3,000 cu yd of material, mostly sand, was carried downstream, and this was almost the only extra cost involved. The simplicity of the entire procedure is evident in Fig. 4, showing the operation as closure was made.

The scheme of building the concrete structures away from the river channel, with the dam founded at the bench level, is relatively unique in the history of hydro power plants. Usually the spillway and the powerhouse are built in the river channel, thus requiring expensive cofferdams and dam foundations in the deepest section of the river. This site, however, with the bend in the river and the sand foundations, was admirably suited to such a scheme, and the result was a substantial saving in cost.

Underwatering.—Lowering the ground water in the excavation (which extended to a maximum depth of about 40 ft below river level) for the powerhouse and spillway was accomplished by a series of deep gravel-wall wells at

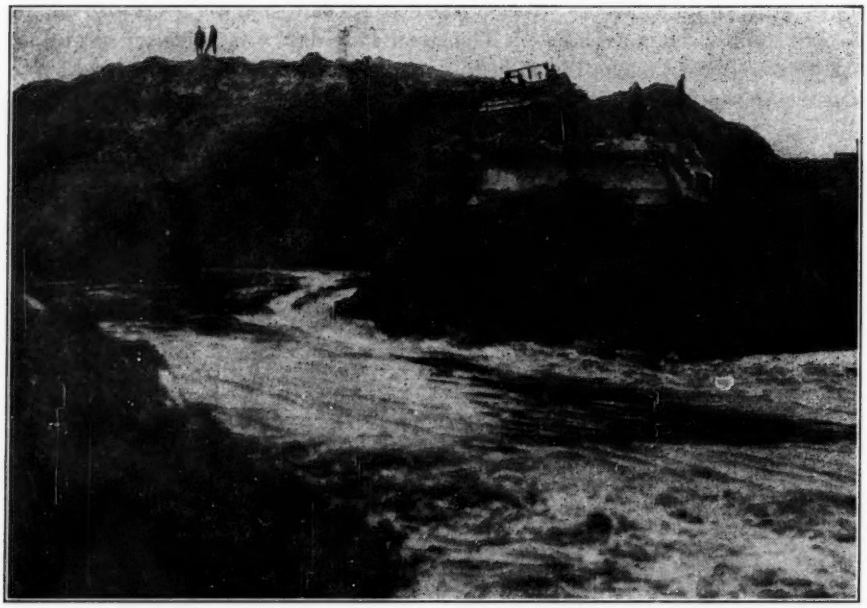

FIG. 4.—CLOSURE OF EARTH DAM

the perimeter of the area, supplemented by well points in the deeper sections. The maximum total pumpage rate was about 11,500 gal per min.

PROCEDURE DURING CONSTRUCTION

Earth Structures.—The earth dam structures consist of the main dam, varying in height from 40 ft to 58 ft above natural ground, and the east and west dikes, both less than 22 ft in height. Fig. 5 shows the main dam, provided with a toe drain to collect the seepage and prevent raveling of the toe. Because of their lower heights, this feature was omitted from the dike structures. The freeboard of the dams is 5 ft above the maximum reservoir level for the design flood and 10 ft above normal reservoir level. In addition, a riprap parapet about 2 ft high is provided as additional wave protection during the period when the reservoir will be surcharged to pass the maximum flood.

The wave protection for the structures consists of a layer of riprap varying in thickness from 1.5 ft to 2.5 ft, depending on the depth of water in front of the dam. This riprap is placed on a filter blanket of gravel 12 in. thick. It extends to a berm at El. 127.0, 3 ft below low water. The material used as

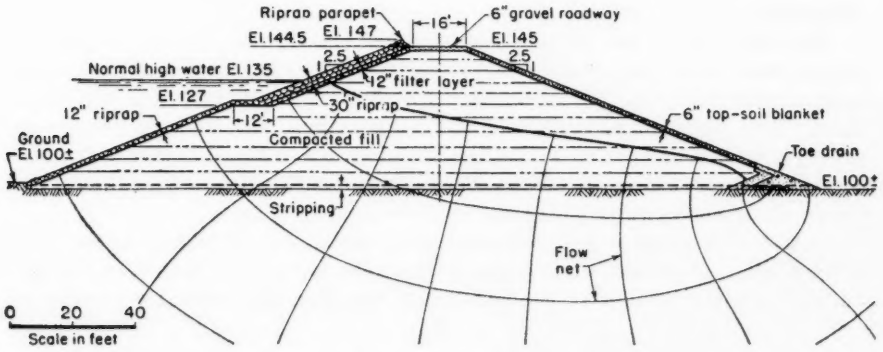

FIG. 5.—CROSS SECTION OF MAIN DAM

riprap has a maximum dimension of 27 in. and a minimum of 4 in., and the blanket material grades from about No. 8 sieve to 3 in. in size. A temporary riprap blanket 12 in. thick was placed on the main dam below El. 127.0, to prevent damage to the fill during the period when the reservoir was filling.

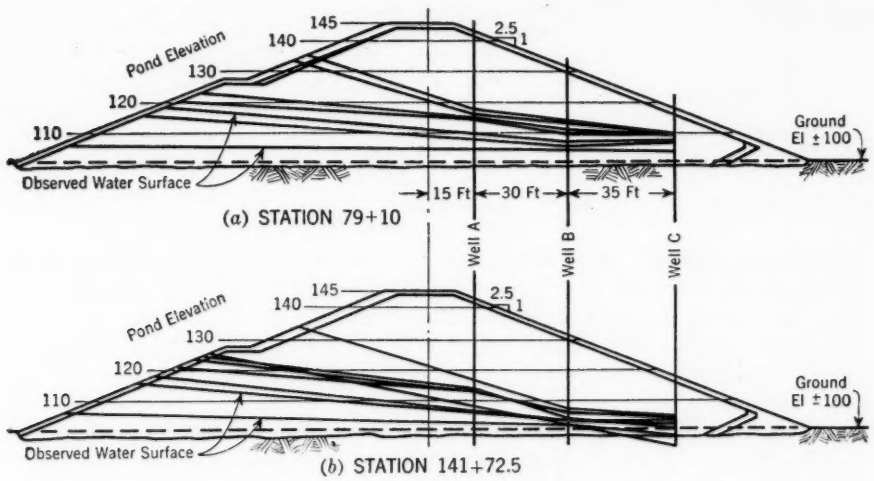

FIG. 6.—PRESSURE LEVELS IN MAIN DAM SECTION

Sand taken from areas adjacent to the structures was used to construct the main dam and dikes. All borrow for the main dam was taken from areas downstream from the dam. This was done in order not to disturb the topsoil upstream from the dam, thereby gaining the maximum use of this topsoil to

reduce leakage through the foundation. Material for the dikes was taken from both sides of the structures, that on the upstream side being taken no closer than 135 ft from the dikes, to leave some topsoil to reduce seepage. Experience with the leakage under the dikes indicates that no difference in underseepage is evident, whether or not borrow was taken from the upstream side of the structures.

The dam and dikes were constructed by placing material in layers not thicker than 12 in. and compacting the material by routing the earth-moving equipment over the surface of the fill. Tests of the completed fill showed dry densities ranging from 97 lb per cu ft to 120 lb per cu ft, with an average density of about 107 lb per cu ft. This method of constructing the dam and dikes was satisfactory and inexpensive. The gradation of the completed fill material is essentially as shown in Fig. 3.

Control of Leakage.—After the diversion of the river through the spillway in December, 1948, the water level in the reservoir rose to El. 105 and then fluctuated with the flow of the river until March, 1949, when the closure of

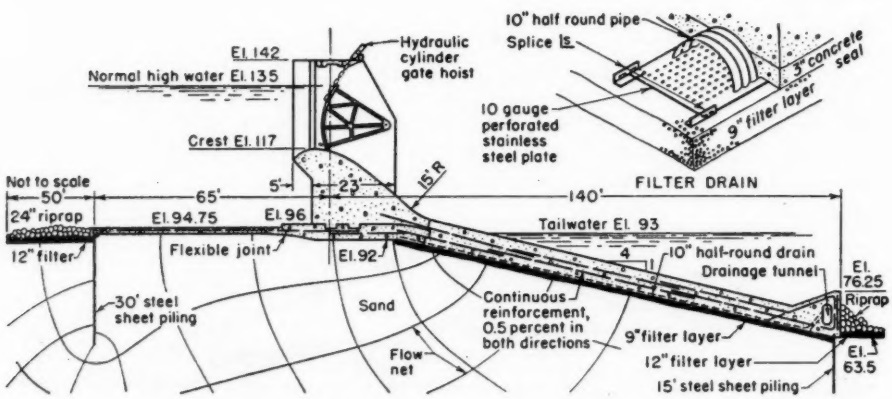

FIG. 7.—CROSS SECTION OF SPILLWAY

the spillway started. The pond was then raised to El. 117, the spillway crest, as rapidly as the river flow would permit, allowing a minimum discharge to keep the hydro plants downstream in operation. In May, 1949, the pond rose to El. 120 and in October to El. 125 (or 10 ft below the full reservoir level), and it remained at this level until the spring of 1950 when the reservoir was filled entirely.

As was expected, considerable leakage developed through and under the dam and dikes. By the latter part of March, 1949, general seepage increased, and boils appeared in a few places along the downstream toe of the dam. The larger boils and those close to the toe of the dam were blanketed with filter gravel to dissipate the flow and permit drainage without movement of sand. Because the structures were compacted to a density considerably higher than natural, the leakage through them was a great deal less than that through the foundation. Well points were installed in several sections of the dam and

in the dikes to observe the seepage line. Fig. 6 shows the observed seepage lines for two of these sections. It can be seen that the top flow line is close to that found from the flow net study of the cross section made prior to construction (Fig. 5).

The leakage through and under the dikes was collected in a ditch located 50 to 75 ft from the toe. Flow net studies prior to construction indicated a total flow of about 108 cu ft per sec from the west dike, using an average coefficient of permeability of $k = 210$ ft per day for both the structure and the foundations. Measurements of the actual flow in the ditch indicated a flow of about 76 cu ft per sec. This agreement is well within the accuracy of measurement of the constants and quantities involved, especially because some of the seepage passed under the ditch and, therefore, was not measured. The east dike leakage was estimated prior to construction to amount to about 79 cu ft per sec, and measurements show an actual flow of about 24 cu ft per sec. The leakage through the main dam section could not be measured but is apparently about as expected.

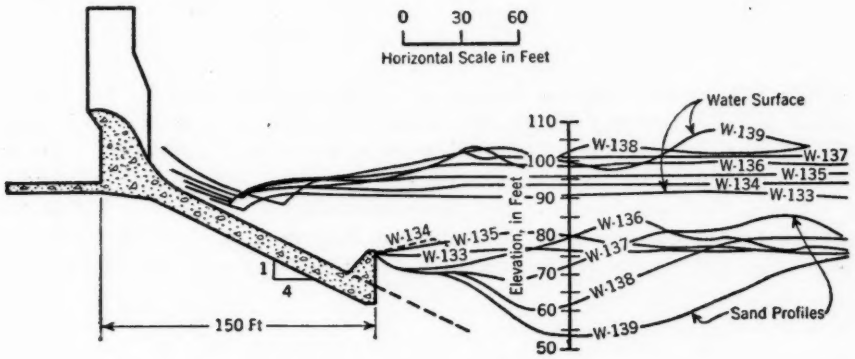

FIG. 8.—MODEL TEST OF SPILLWAY

When the reservoir was raised to full height in the early spring of 1950, considerable underground seepage developed outside of the original drainage ditch at the toe of the dike. The land outside of the dike is very flat, consequently the underseepage raised the ground-water table to the surface, forming shallow pools in low spots as far as a mile from the dike. To alleviate this condition, a second ditch was required, deeper than the original and located 200 to 500 ft from the dike. This satisfactorily intercepted the underseepage and dried up the surrounding land.

Spillway Construction.—The spillway contains 16 radial (tainter) gates, each 30 ft long and 18 ft high, designed to pass a flood of 134,000 cu ft per sec at normal reservoir elevation (135 Petenwell Datum) and capable of passing 200,000 cu ft per sec with a 5 ft surcharge on the reservoir. The maximum flood of record at this point in the river is about 95,000 cu ft per sec, having occurred in June, 1905. The drainage area of the river above the project is 5,800 sq miles.

The type of hydraulic design chosen for the spillway (Fig. 7) utilizes the principle of the sloping apron to provide the hydraulic jump. The foundation of the ogee section is at a very high level, just a few feet below the natural ground surface, with the sloping apron extending approximately 118 ft downstream to an elevation of 63.5, some 30 ft below the river level. Comparisons of the cost of this structure with one having a conventional horizontal apron,

TABLE 1.—SPILLWAY MODEL TEST DATA

Run	Gate opening (feet)	Elevation (in Feet)		Flow (Cu Ft per Sec)	
		Headwater	Tailwater	One gate	Sixteen gates
(1)	(2)	(3)	(4)	(5)	(6)
W-133	1.60	134.9	91.3	935	14,960
W-134	3.00	135.0	94.1	1,750	28,000
W-135	6.02	135.0	97.5	3,570	57,120
W-136	8.87	134.9	99.1	5,120	81,920
W-137	12.06	135.2	100.7	7,140	114,240
W-138	full	135.0	102.4	10,030	160,480
W-139	full	138.0	102.3	13,500	216,000

with or without baffle blocks, showed considerable saving in favor of the sloping apron design. Building the structure away from the river permits the foundation of the gravity section of the spillway to be about 10 ft higher than it would have been if built in the river bottom, with a considerable saving of concrete.

A series of model tests of the Petenwell spillway was carried out at the Hydraulic Laboratories of the University of Wisconsin at Madison. A model of a single bay was placed in a glass-walled flume, and tests were made with

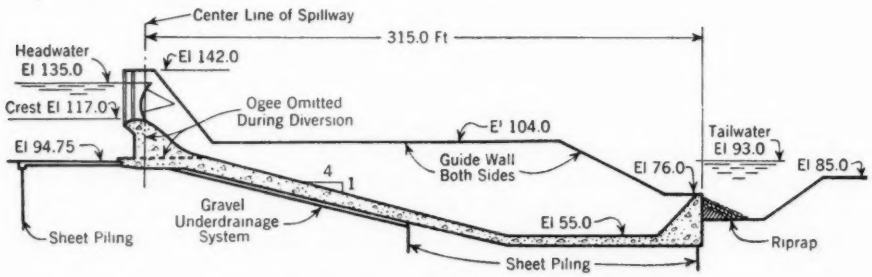

FIG. 9.—CROSS SECTION OF REGULATING BAY

different lengths of apron, various sill shapes and with and without baffles on the apron. The scale of this model was 1 to 25. The scour pattern with various flow and tailwater conditions was recorded for each variation of the model. Fig. 8 shows the scour patterns observed for various gate openings. The scour was continuous during this test; that is, the gate was opened a small amount, and a ½-hr run was made; then, without changing the sand configuration in the model, the gate was opened further, and another run was made.

The test was continued in this way until the gate was wide open and the scour in the model had stabilized. Table 1 contains pertinent data on the test run shown in Fig. 8.

Fig. 8 shows the hydraulic action of this type of design. The sloping apron provides an increasing depth of tailwater as the overflow sheet proceeds downstream. The length of the apron required is such that the hydraulic jump will occur on the apron for all conditions of overflow and corresponding tailwater elevations. The sill at the end of the apron deflects the water upward and sets up a large ground roller wave that causes a strong upstream current along the bottom just downstream from the sill. This is the fundamental

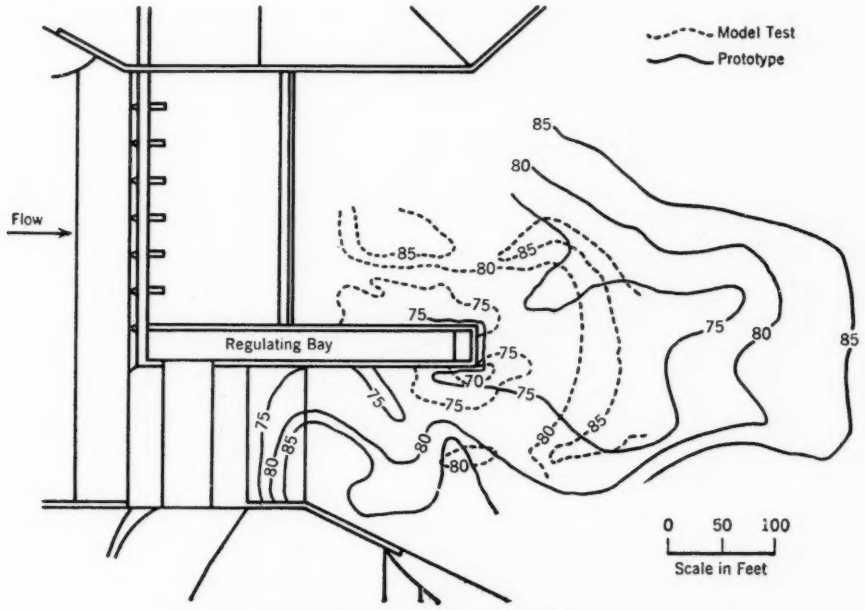

FIG. 10.—REGULATING BAY SCOUR PATTERN

point in the design. The upstream current deposits material against the sill rather than scouring at that point. Both the length of the sloping apron and the shape and height of the sill were determined from the model studies.

Each of the models tested was operated for one run with the tailwater corresponding to an elevation about 5 ft below the normal river elevation for the corresponding flow. This was done to insure that satisfactory operation would be obtained if (*a*) the tailwater rating curve were not accurate and (*b*) if any tailwater degradation were to occur.

The spillway structure, as shown in Fig. 7, is founded directly on the sand, on a concrete slab 4 ft thick with $\frac{1}{2}$ of 1% continous reinforcing steel in both directions. The reinforcement runs through all joints. The ogee monoliths resting on the foundation slab are 35 ft long, separated by a contraction joint located in the middle of each gate opening. The slab under the gravity

(ogee) section is tied into an upstream apron slab that has a steel sheet pile cutoff 30 ft deep at its upstream edge. The downstream sloping apron is also monolithic with the slab under the ogee section and is provided with a sheet pile cutoff 15 ft deep at its downstream edge. Sheet pile cutoffs are provided at each end of the structures, thus completely confining the sand in the foundations. All slabs have continous reinforcing of 0.5% in both directions running through all joints.

The drainage provisions under the downstream sloping apron are of great importance because it is essential that no pressure from headwater be permitted to act against the apron. A drainage layer 9 in. thick was placed under the entire apron, as shown in Fig. 7, designed to act as a filter and prevent the foundation sand from being carried through. The leakage into the filter drain is collected in half-round pipes spaced 17.5 ft on centers at the bottom of the

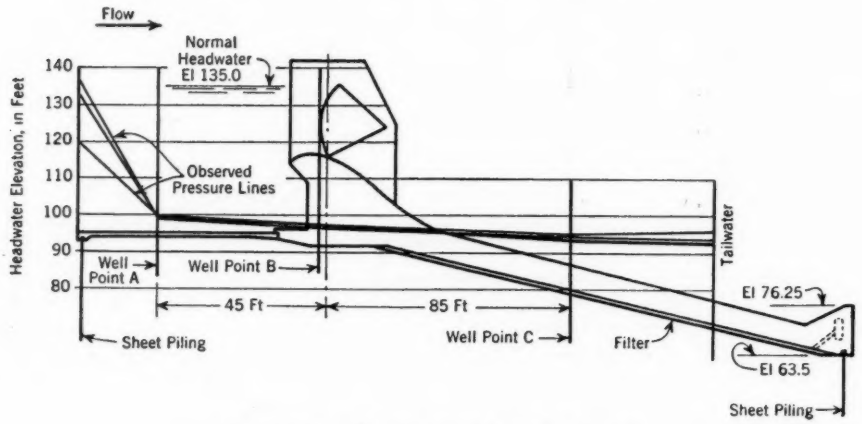

FIG. 11.—PRESSURE LEVELS IN SPILLWAY SECTION

apron slab and then goes into 2 ft by 6 ft drainage tunnel in the toe of the apron. This tunnel discharges into the tailrace through openings in the training walls at each end.

The details of the joint between the ogee section and the upstream slab are also shown in Fig. 7. This joint must remain watertight; otherwise the effect of the slab is lost entirely. The joint was so designed that a positive bearing pressure exists at all times on the horizontal face. This pressure is a maximum before the water pressure acts against the ogee section and is reduced when the water load comes on. However, the design maintained a residual pressure of about 700 lb per linear ft after all the water load was against the dam. A poured hot rubber-asphalt seal was also placed at the joint to provide additional security against leakage.

The upstream, or heel, apron provides additional resistance against sliding of the structure. It is watertight, and therefore a net vertical force acts downward on the slab. This force is equal to the difference of the downward pressure of the full head and the upward pressure under the slab, which is progressively reduced in the downstream direction by friction loss in the founda-

tion sand. The maximum calculated bearing pressure under the spillway is about 2,500 lb per sq ft, and the sliding factor is 0.4.

The sand bed of the river is so erodible that the spillway gates in the main section must all be opened at the same time and to an equal opening. If this is not done, eddies that would soon scour deep holes in the sand would be set up downstream from the spillway and jeopardize the safety of the structure. However, opening all the gates at once is not a satisfactory method

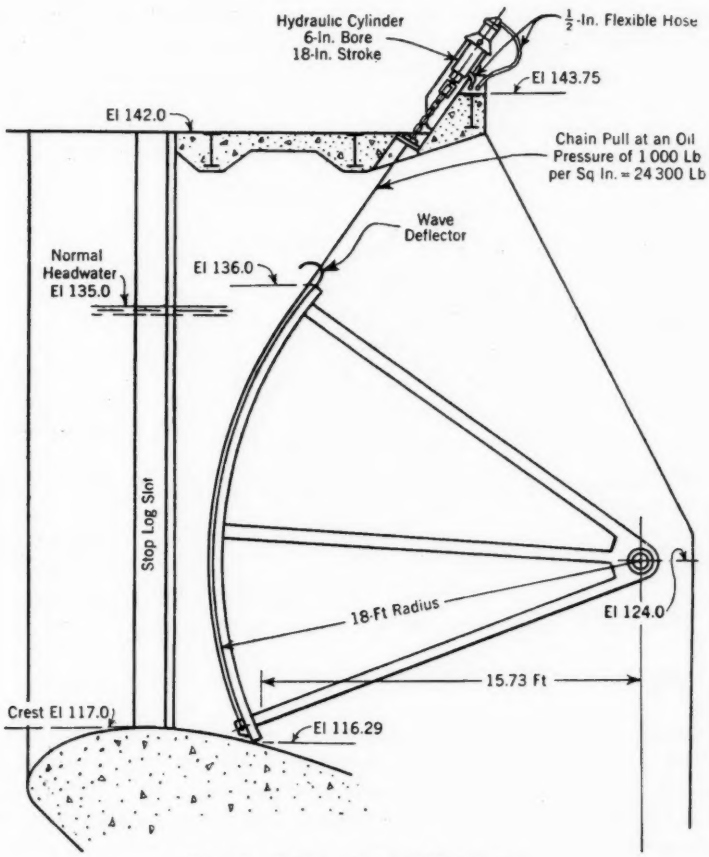

FIG. 12.—DETAIL OF GATE MECHANISM

of operation to pass flows only slightly in excess of the plant capacity. Ice or trash could not be passed through the dam since opening all gates wide enough to do this would waste so much water. As a result of these factors it was decided to provide the necessary stilling basin for one gate so that it could be opened by itself. This bay is called the regulating bay, and the section is shown in Fig. 9.

Because the flow through one bay causes so much lower tailwater than caused by opening the other 15 gates correspondingly, it was necessary to

extend the sloping apron farther out for the regulating bay than for the remainder of the spillway. It was also necessary to confine the flow for a sufficient distance so the major portion of the energy of the water is dissipated by the time the water discharges to the sandy river bed. The slope of the retaining walls at the downstream edge causes a fanning out of the flow as it leaves the stilling basin and prevents undesirable eddies at the corners. The design of the regulating bay was also verified by model tests. Fig. 10 shows the contours of the scour pattern as observed in the model tests.

Diversion was carried out by omitting the ogee section of the regulating bay and permitting the river to pass through this bay. During the spring of 1949 a flow of more than 10,000 cu ft per sec was passed through this structure. This is considerably more flow than the 8,000 cu ft per sec that can pass through the gate of this bay with the ogee section in place, and so was a good test of the prototype. The scour caused by this condition was measured

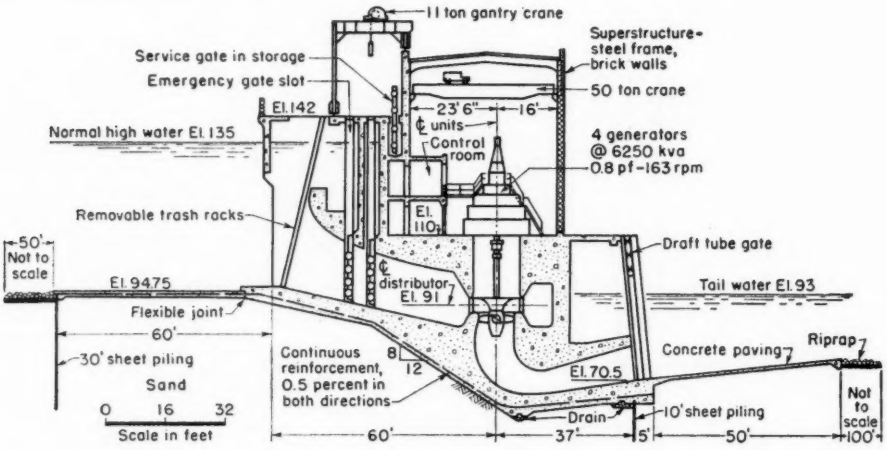

Fig. 13.—Cross Section of Powerhouse

by sounding and observations by a diver, and these contours are also shown in Fig. 10. Comparison of the contours with the model test scour pattern indicates the correlations between model and prototype to be satisfactory.

To determine the actual water pressure under the spillway structure and thus obtain information as to the adequacy of the upstream sheet piling cutoff and apron, well points were placed under the upstream slab at three sections of the structure. Pipes from the well points were brought up in the piers to permit easy observation. Fig. 11 shows the observed pressures at one of these sections (the others were similar) for various reservoir elevations. The large pressure drop between headwater pressure and the first well point (Point A on Fig. 11) shows that the sheet pile cutoff is functioning very well. It should be noted that this pressure drop is greater than would be anticipated from a flow net drawn on the basis of a coefficient of permeability equal in all directions. It indicates that the coefficient of permeability in the vertical

direction is smaller (less pervious) than in the horizontal direction, a condition to be expected in an alluvial bed of sand. This condition also adds to the factor of safety against sliding.

Spillway Gate Hoists

A novel gate hoist mechanism (Fig. 12) was used to insure uniform operation of the 15 gates in the main spillway section. Each gate is raised by a pair of hydraulic cylinders, all of which are operated from a single pumping system. The maximum stroke of the cylinders is 18 in., resulting in a maximum lift of the gates of about 12 in. The gates are dogged after being lifted 12 in., and the hook is re-engaged to permit the next lift. This operation takes about 30 min for all gates. The flow permitted by the first 1-ft opening of all gates is about 9,000 cu ft per sec.

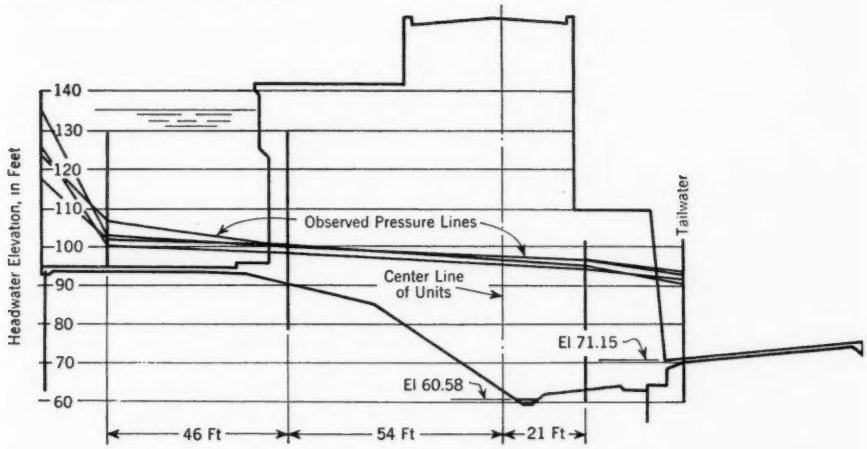

Fig. 14.—Pressure Levels in Powerhouse Section

This hoisting system proved less than half as expensive as any other system that would permit uniform opening of all the gates at the same time, and it is operating satisfactorily. Some adjustment of the valving was required to balance the system, but otherwise no difficulties have been experienced. Also the gates can be raised individually if conditions require. The gate on the regulating bay is operated by a conventional motor operated hoist.

Powerhouse

Fig. 13 shows a cross section of the powerhouse. The same principles of foundation design were applied in this structure as in the spillway. A continous slab reinforced with 0.5% of steel in both directions forms the foundation. This foundation slab is integral with the spillway foundation slab, so in effect both structures are on a continous mat that is bounded on all sides by steel sheet piling. The drainage provisions are shown on the section.

The powerhouse substructure and the intake represent a single monolith over 140 ft long without contraction joints. Shrinkage cracks were held to a

minimum by reducing the size of pours and building alternate units ahead of others. The intake bulkhead wall separating the generator room from headwater also contains 0.5% of steel in the horizontal direction. Its watertightness has proved very satisfactory.

Vertical-lift wheeled gates are provided for the intake. The gates are of all welded construction, and the wheels are equipped with roller bearings to reduce hoisting effort and eliminate by-pass, or filler, valves. The trashracks of all welded construction are built in sections that are self supporting and can be removed entirely, thus eliminating any fixed submerged members inaccessible to inspection.

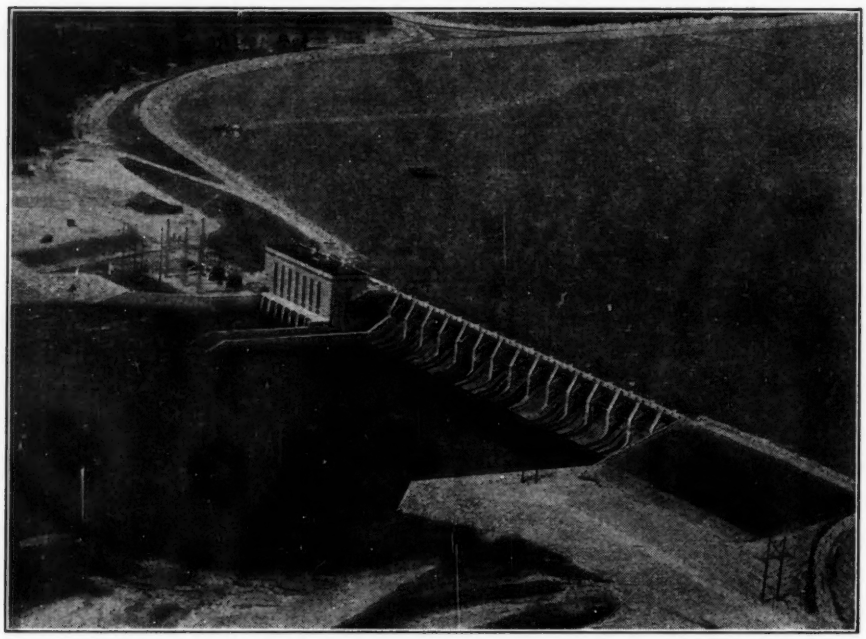

FIG. 15.—AERIAL VIEW OF COMPLETED PROJECT

Water pressure readings have been taken under the powerhouse, similar to those for the spillway and main dam. The results of readings at a typical section are shown in Fig. 14. The large pressure drop caused by the sheet piling cutoff is clearly evidenced. Although the drainage provisions under the draft tube do not appear to be quite as effective as the drainage layer under the spillway apron, the pressures are well within the design limits, and observations of the flow from the drains indicate they function adequately.

In the case of a river flowing on sand as this one does, it was anticipated that the river levels below the dam would be lowered considerably after its completion because the reservoir traps all the bed load in the river and a new bed load will be picked up by the river after passing the dam. The turbines are set low to provide for this anticipated degradation.

Settlement records of the spillway and powerhouse have shown that very little, if any, settlement has occurred. The load spreading effect of the highly reinforced continuous foundation slab combined with the relatively low bearing pressures would lead to expectations of only insignificant settlements.

An aerial view of the completed project in operation is given in Fig. 15.

Conclusion

The successful construction and operation of this project has shown that water-retaining structures of considerable size can be built on sand foundations, providing adequate precautions to confine the sand and to handle the underseepage are incorporated in the design. The successful performance of the spillway to date indicates that the sloping apron type of design prevents harmful erosion of the sand in the river bed at a relatively low cost.

Acknowledgments

The powerhouse and spillway were constructed by forces of the Wisconsin River Power Company under the direction of W. F. Thiele, Executive Vice President, and Ralph J. Kutchera, Jun. ASCE, project manager. Foley Bros.-Marsch-Peterson were the contractors for the earth dams.

Arno T. Lenz, Assoc. M. ASCE, of the University of Wisconsin directed the model tests of the spillway.

The Harza Engineering Company was the consulting engineer on this work, which was done under the direction of L. F. Harza, M. ASCE, and the writer. C. E. Wattles was resident engineer on construction.

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